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EAST ST. LOUIS VETERANS MEMORIAL BRIDGE

By A. L. R. Sanders, M. ASCE

CONSTRUCTION AND STRUCTURAL DIVISIONS

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Technical Division	*Proceedings-Separate Number*
Air Transport	42, 43, 48, 52, 60, 93, 94, 95, 100, 103, 104, 108, 121, 130, 148 (Discussion: D-23, D-43, D-75)
City Planning	58, 60, 62, 64, 93, 94, 99, 101, 104, 105, 115, 131, 138, 148, 151, 152 (Discussion: D-16, D-23, D-43, D-60, D-62, D-65, D-86)
Construction	43, 50, 55, 71, 92, 94, 103, 108, 109, 113, 117, 121, 126, 130, 132, 133, 136, 137, 145, 147, 148, 149, 150, 152, 153 (Discussion: D-3, D-8, D-17, D-23, D-36, D-40, D-71, D-75, D-92)
Engineering Mechanics	122, 124, 125, 126, 127, 128, 129, 134, 135, 136, 139, 141, 142, 143, 144, 145 (Discussion: D-24, D-33, D-34, D-49, D-54, D-61, D-96, D-100)
Highway	43, 44, 48, 58, 70, 100, 105, 108, 113, 120, 121, 130, 137, 138, 144, 147, 148, 150, 152 (Discussion: D-XXVIII, D-23, D-60, D-75)
Hydraulics	107, 110, 111, 112, 113, 116, 120, 123, 130, 134, 135, 139, 141, 143, 146, 153 (Discussion: D-70, D-71, D-76, D-78, D-79, D-86, D-92, D-96)
Irrigation and Drainage	97, 98, 99, 102, 106, 109, 110, 111, 112, 114, 117, 118, 120, 129, 130, 133, 134, 135, 138, 139, 140, 141, 142, 143, 146, 148, 153 (Discussion: D-40, D-44, D-47, D-57, D-70, D-71, D-76, D-78, D-80, D-86, D-87, D-92, D-96)
Power	48, 55, 56, 69, 71, 88, 96, 103, 106, 109, 110, 117, 118, 120, 129, 130, 133, 134, 135, 139, 141, 142, 143, 146, 148, 153 (Discussion: D-38, D-40, D-44, D-70, D-71, D-76, D-78, D-79, D-86, D-92, D-96)
Sanitary Engineering	55, 56, 87, 91, 96, 106, 111, 118, 130, 133, 134, 135, 139, 141, 149, 153 (Discussion: D-29, D-37, D-56, D-60, D-70, D-76, D-79, D-80, D-84, D-86, D-87, D-92, D-96)
Soil Mechanics and Foundations	43, 44, 48, 94, 102, 103, 106, 108, 109, 115, 130, 152 (Discussion: D-4, D-XXVIII, D-7, D-43, D-44, D-56, D-75, D-86)
Structural	117, 119, 121, 122, 123, 124, 125, 126, 127, 128, 129, 132, 133, 136, 137, 142, 144, 145, 146, 147, 150 (Discussion: D-51, D-53, D-54, D-59, D-61, D-66, D-72, D-100)
Surveying and Mapping	50, 52, 55, 60, 63, 65, 68, 121, 138, 151, 152 (Discussion: D-60, D-65)
Waterways	41, 44, 45, 50, 56, 57, 70, 71, 96, 107, 112, 113, 115, 120, 123, 130, 135, 148 (Discussion: D-8, D-9, D-19, D-27, D-28, D-56, D-70, D-71, D-78, D-79, D-80)

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

EAST ST. LOUIS VETERANS MEMORIAL BRIDGE

By A. L. R. Sanders,[1] M. ASCE

Synopsis

Many of the important problems that were encountered in the design of the East St. Louis Veterans Memorial Bridge—said to be the longest cantilever span across the Mississippi River—are described in this paper. Solutions evolved by the engineering consultants and steel fabricators are discussed and described.

Specifications governing the design of the structural members on the bridge superstructure are given to indicate a number of modifications in the standard design code. Various design details, such as shoes, floor beam hangers, anchorages, lateral bending in floor beams, and trestle construction are described and illustrated.

Introduction

Although the East St. Louis Veterans Memorial Bridge is believed to be the longest cantilever span crossing the Mississippi River, it is the intent of this paper to emphasize a few of the important details of the bridge, rather than the routine engineering involved in the design and supervision of construction of a cantilever highway bridge.

The project, consisting essentially of a river crossing connected to a grade separation by a hydraulic fill embankment 1,700 ft long, is approximately 1.5 miles long. The river crossing (see Fig. 1) consists of one 964-ft channel span, two 470-ft anchor spans, two 213-ft deck truss spans, two 120-ft girder spans, 1,440 ft of steel trestle and, at the western end, a 160-ft paved approach fill within retaining walls. The 964-ft channel span includes a 324-ft suspended span.

The grade separation consists of one 250-ft through truss span, 1,000 ft of steel trestle, and, at the eastern end, a 550-ft paved approach fill. The three main river spans required 5,150 tons of fabricated steel, and have a total length of 1,904 ft.

Note.—Written comments are invited for immediate publication; the last discussion should be submitted by **March 1, 1953.**

[1] Partner and Chf. Engr., Hazelet & Erdal Cons. Engrs., Chicago, Ill.

Substructure

Design of the substructure offered no particular problems since all main piers are founded on limestone rock. Pier No. 9 (Fig. 1) was constructed inside a steel sheet-pile cofferdam. Caissons, sunk by dredging through wells in the caissons, were used in the construction of Pier Nos. 10 and 11. It was necessary to sink the caisson at Pier No. 11 through 28 ft of water and 65 ft of coarse river sand containing some silt and gravel. The caissons were finally seated and

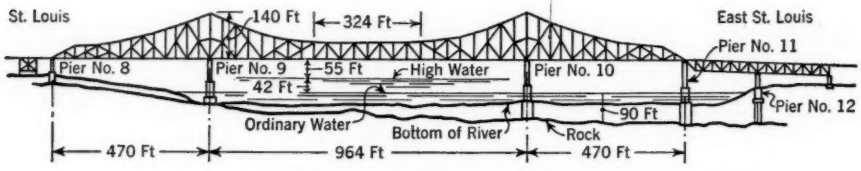

Fig. 1.—Elevation of East St. Louis (Ill.) Veterans Memorial Bridge

sealed under air. The deepest pier (Pier No. 11) required 48 lb-pressure, which is nearly the maximum pressure under which men are permitted to work. Twin circular caissons were used in constructing Pier No. 11, which was designed for much lighter loads than Pier No. 10, where a single caisson was used.

Superstructure Specifications

The design of the span was based on the specifications of the American Association of State Highway Officials (AASHO),[2] except for certain modifications, the most significant of which are concerned with (1) the design of the main compression members and (2) wind loads.

1. *Design of Main Compression Members.*—To provide adequate local buckling strength in main compression members, the recommendations of L. T. Wyly, M. ASCE, in regard to plate thickness,[3] were adopted in lieu of the requirements of the AASHO.

The thicknesses (t) of plates and outstanding flanges and legs of angles were limited to not less than—

$$t_a = \frac{b}{\sqrt{\frac{26{,}250{,}000}{s}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

—for the case of plates supported on two edges, such as webs and cover plates of truss members, and to—

$$t_w = \frac{b}{\sqrt{\frac{2{,}187{,}500}{s}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1b)$$

—for the case of plates supported on one edge, such as the section of perforated cover plates along the perforation and for the case of outstanding legs of angles. On main members and bracing members having low unit stresses, the minimum

[2] "Standard Specifications for Highway Bridges," adopted by the Am. Assn. of State Highway Officials, Washington, D. C., 1944.

[3] "Rational Design of Sections for Short Compression Members of Steel," by L. T. Wyly, *Bulletin No. 467*, Am. Ry. Eng. Assn., Chicago, Ill. June-July, 1947, p. 3.

thickness was set at $b/60$ for plates supported on two edges and $b/16$ for plates supported on one edge.

In Eqs. 1, s is the computed axial unit stress in a member and b is the width, defined as follows:

a. Plate width is the distance from the unsupported edge (the edge of a hole in a perforated plate) to the supported edge for plates supported at one edge only, the total width between edges for plates supported along two opposite edges only, and half the width between edges for plates supported along the two longitudinal edges and along the center.

b. The width of an outstanding leg is the total outstanding leg of an angle and half the flange width of rolled beams unsupported along the outside edges.

For carbon steel under an assumed unit stress of 15,000 lb per sq in., the minimum thickness for a plate supported on two edges would be 1/42 of plate width. For silicon steel at 20,000 lb per sq in., the ratio would be 1/36 of plate width. In the case of plates supported at one edge and in the case of the outstanding legs of angles, the minimum thickness at 15,000 lb per sq in. and 20,000 lb per sq in. would be 1/12 and 1/10.5, respectively.

The "width of plate" intended under the design specifications is the total width. The AASHO specification requires the thickness of plates making up the section to be a function of the distance between the nearest rivet lines, or the roots of flanges of rolled sections. In the case of main webs, the function is 1/32 for carbon steel and 1/28 for silicon steel; for solid cover plates and diaphram plates, the functions are 1/40 and 1/36, respectively, and in the case of perforated cover plates, the function is 1/50 for carbon steel and 1/45 for silicon steel.

In general, the web plates on the East St. Louis Veterans Memorial Bridge were 26 in. wide and the rivet lines were 18½ in. on center. The design specification would require 0.62-in. web plates, if carbon steel were used at 15,000 lb per sq in., whereas the AASHO specifications would require 0.58-in. plates. Generally, the solid cover plates and the perforated cover plates were 22½ in. wide with rivets at 16 in. on center. The design specifications would require 0.54-in. cover plates, if carbon steel were used at 15,000 lb per sq in., whereas the AASHO specifications would require 0.40-in. plates.

The design specifications, however, permit a reduction in plate thickness with a reduction in unit stress, and all plates entering into the make-up of the member are treated on the same basis, which seems to be the logical procedure. In most of the chord and web compression members, the loads were so large that the sections chosen were well above the minimum thickness.

2. *Wind Loads.*—The following requirements for wind loads were substituted for those specified by the AASHO in those spans that were more than 250 ft long, all wind loads being considered horizontal:

Transverse Wind Loads, Bottom Laterals.—A moving horizontal load equal to 30 lb per sq ft on 1½ times the area of the structure as seen in elevation, plus 200 lb per lin ft with 25% increase in normal unit stresses.

Transverse Wind Loads, Top Laterals.—Top laterals shall be designed for a moving wind load of 30 lb per sq ft at normal unit stresses.

Transverse Wind Loads, Chord Members.—When chord members are part of the portal system, they shall be designed for dead load plus live load plus impact, plus a moving horizontal load of 30 lb per sq ft with 25% increase in normal unit stresses. All other chord members shall be designed for dead load plus live load plus impact, plus a moving horizontal load of 15 lb per sq ft with a 25% increase in normal unit stresses.

Longitudinal Wind Loads.—Longitudinal wind loads equal to 70% of the specified transverse wind load shall be applied to the bridge. Such wind load shall not be assumed to act simultaneously with the longitudinal forces specified as a result of braking loads.

Design and Details of the Superstructure

Main Members.—All elements, except most of the hangers, were boxed members in which the cover plates had oblong cutouts, at regular intervals, to provide access for riveting and painting. These holes were 10 in. by 20 in., except at panel points, where in some cases it was necessary to make them 12 in. by 24 in. to allow a man to enter the member during riveting. In the latter case the member was reinforced to compensate for the additional sectional area removed.

The AASHO specification requires the removal of at least $\frac{1}{4}$ in. of metal by milling, chipping, or grinding in the case of flame-cut edges of silicon steel. This is an extremely costly requirement.

The fabricator was permitted to flame-cut the holes, providing the edges were annealed simultaneously by a gas flame. This procedure had previously been used on two other large bridges in which extensive tests were made before the procedure was adopted by the engineers.

Bridge Shoes.—On the main piers at panel point L14, the original design contemplated the use of a shoe in which the members entering the joint were joined by large pins. A study indicated that the bearing pressure on these pins would be so great that rotation could hardly be expected to take place. Jonathan Jones, Hon. M. ASCE, suggested the use of a rocker type of shoe. Study of this problem by both the Bethlehem Steel Company, Bethlehem, Pa., and the consultants resulted in the development of the rocker shown in Fig. 2. This shoe is subjected to a total load of 4,840,000 lb. The radius and face width were proportioned on the basis of a formula for large rollers proposed by Wilbur M. Wilson, Hon. M. ASCE.

$$w = (12{,}000 + 80\,D)\,\frac{s_y - 13{,}000}{23{,}000} \qquad (2)$$

in which w is the allowable load per linear inch; D is the diameter of the roller, in inches; and s_y is the yield point of material in pounds per square inch.

"Mayari R" steel was used for the $7\frac{1}{2}$-in. slab on the top of the bearing and the 9-in. slab and $1\frac{1}{2}$-in. sole plate on the truss. The material conformed to the American Society for Testing Materials (ASTM) specification A242, except that it had a specimen yield point of about 45,000 lb per sq in.

The remainder of the shoe was fabricated from material meeting ASTM specification A7, except that physical tests were waived and the carbon and

manganese contents were specified to lie within the range of from 0.15 to 0.25 and 0.40 to 0.60, respectively, by ladle analysis. Killed steel was used. The physical tests were waived since it is known that specimens cut from thick slabs will not test as high as those from ordinary structural shapes. Lower designing unit stresses were used to compensate for the lower yield point.

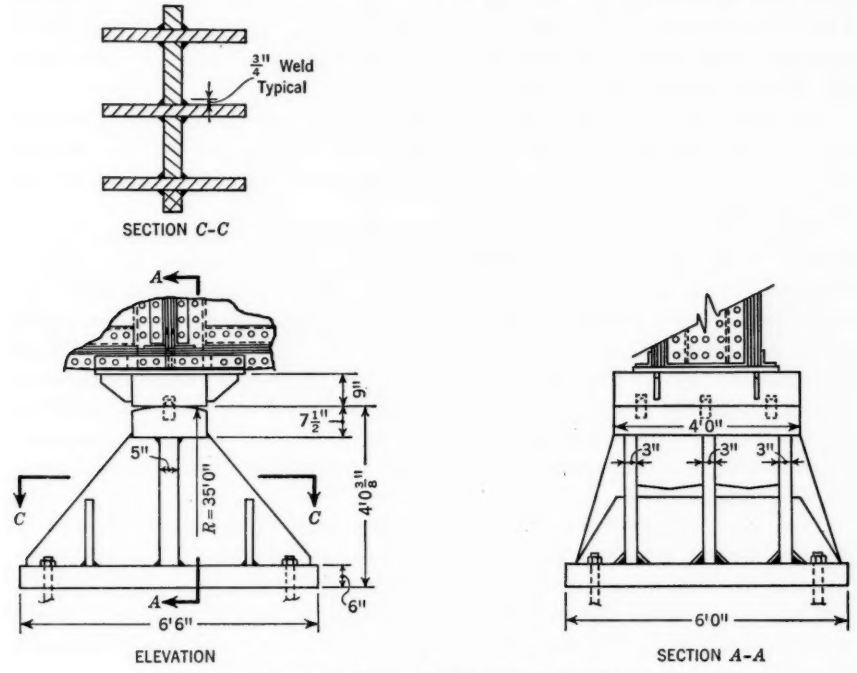

FIG. 2.—DETAIL OF ROCKER BEARING AT PANEL POINT L14

For material having a yield point of 45,000 lb per sq in., a 70-ft-diameter roller would have a permissible line bearing of 110,000 lb per lin in. The actual line bearing, based on a net width of 40.375 in., is 120,000 lb per lin in. Tests conducted by Mr. Wilson indicated that the critical load was 50% greater than the permissible load given by Eq. 2, and for the case considered here would be approximately 165,000 lb per lin in.

The width (b_c) of the area in contact, as a result of flattening, can be determined by the Hertz formula:

$$b_c = 0.0004 \sqrt{w \times D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in this case b_c is 4 in. On that basis, the average unit pressure is approximately 30,000 lb per sq in. The natural tendency in constructing the shoe would be to place one main web transverse to the center line of the truss. To do so, however, would require excessively large welds attaching the longitudinal stiffening plates to the web. The shoes were constructed using three 3-in. longitudinal webs with short transverse sections of 5-in. material between the webs. This arrangement greatly reduced the shear on the vertical welds.

The shoes, upon completion of the welding, were normalized in accordance with the 1946 American Society of Mechanical Engineers (ASME) code for the fusion welding of unfired pressure vessesls.

Floor Beam Hangers.—Considerable time was devoted to the design of the floor beam connections and to the design of those truss members commonly called floor beam hangers. The rotation of the ends of floor beams (particularly in structures having wide roadways) caused by the deflection of the beams under the dead load of the floor system, plus live and impact loads, porduce high bending stresses in the floor beam hangers.

Mr. Wyly shows[4] that, unless a correction is made in the usual analysis of the frame formed by a floor beam, the floor beam hangers, and the top strut or portal frame, stresses considerably in excess of those calculated by the usual analysis should be anticipated.

Hangers in the plane under discussion are displaced laterally at the top of the floor beam, as well as rotated through the same angle as the end rotation of the floor beam since floor beams are usually relatively deep compared to the width of floor beam hangers. The stress resulting from this translation might readily be overlooked in the design of the hanger. The end connections of all floor beams were canted out at the top through an angle approximately equal to the end rotation of the floor beam caused by full dead load plus one-half live load and impact. This induced an erection stress of opposite sign in the hanger, as a result of bending, approximately equal to that resulting from full dead, live, and impact loads.

Anchorage.—Short links were used at the ends of the anchor spans to transmit the end reactions to the anchor piers. Short links, 2 ft 9 in. center to center of pins, were not objectionable since an overlapping finger type of joint was used across the roadway and no harm would be done by a slight unevenness caused by the rise and fall resulting from the rotation of the short links as the anchor spans expanded and contracted.

Lateral Bending in Floor Beams.—The steel company fabricated the stringers to the theoretical panel length and cambered the trusses so that, under full dead load, the trusses would assume the design proportion—that is, have the grade and panel length shown on the stress sheet. During erection, the ends of the floor beams in many cases must be bent laterally, in order to connect them to the trusses because of the change in length of the bottom chord members in cambering. This bending was relieved to a considerable extent by resting the stringers on shelves provided on occasional floor beams, rather than framing them into the floor beam. For the concrete floor slab a pour sequence was selected that would not aggravate the lateral bending in the floor beams. It was also realized that, after all construction was finished, the lateral bending in the floor beams would be produced by the passage of live load over the structure. The lateral flexibility of the top flanges of the floor beams was increased by painting their top surfaces with an asphalt paint (so that there would be no bond between concrete and steel) from the truss connection to the

[4] "Two Problems in Bridge Design," by L. T. Wyly, *Bulletin No. 454*, Am. Ry. Eng. Assn., Chicago, Ill., September-October, 1945, p. 27.

first interior stringer. The concrete slab was carefully formed to permit lateral bending for the length thus painted.

Trestle Construction.—Many engineers visiting the project were interested in the trestle construction. In general, the trestles were a series of four continuous spans separated by braced tower spans. In order to reduce to a minimum the moment in the longitudinal direction on the pedestal piers, the bents were constructed with rockers at the base of the columns and under each stringer, where they passed over the floor beams. Thus, each bent between braced towers was free to rock back and forth. One end of the continuous construction was fixed to a braced tower and the other end was left free to slide on the next braced tower.

Acknowledgments

The engineering board charged with the bridge design consisted of the late Charles A. Ellis, M. ASCE, and the late George A. Maney, M. ASCE; Mr. Wyly; and Hymen Shifrin, M. ASCE, of the firms of Horner & Shifrin and Hazelet & Erdal, consultants. The three main spans were fabricated and erected by the American Steel Company.

The writer is indebted to Mr. Wyly for his review of the design and his helpful suggestions.

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